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SHEAR-BOND STRENGTH OF STUDDED STEEL DECK SLABS

by

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SUMMARY

The shear-bond strength of composite deck slabs was increased by 8% to 33% by studs located at the end of the span. A design approach utilizing a linear regression for nonstudded specimens also appeared reasonable for the studded slabs.

INTRODUCTION

The use of cold-formed steel decking as reinforcement for concrete floor slabs has increased markedly in the past twelve to fifteen years. This kind of composite floor system has two primary advantages: the ability of the steel deck to provide formwork during the casting stages of the concrete and the ability to serve in composite action as tension reinforcement under positive bending. Composite shear transfer devices such as deck embossments or transfer wires provide restraint for horizontal shear which develops at the deck-to-concrete interface. Many other advantages in addition to those mentioned above are given in Ref. [7].

Design recommendations for vertical loads applied to formed metal deck composite slabs have been developed at Iowa State University [8]. The design is controlled primarily by one-way behavior; that is, the relatively large bending stiffness of the slab parallels the longitudinal direction of the deck. Previous research at Iowa State University [6-10] has resulted in design equations [8] for predicting the load capacity of one-way steel-deck-reinforced composite slabs without end-span studs. The predominant mode of failure was found to be shear-bond [10]. The design equation for shear-bond capacity prediction was based on a modification of Eq. (11-6) of the American Concrete Institute (ACI) Code [1].

For steel deck slabs that have end-span studs, two types of composite behavior need to be considered. A general view of such a floor slab system with both composite deck and studs is shown in Fig. 1. Past research on composite steel deck slabs has not considered the effect of stud end restraint on shear-bond behavior. Also, past

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research and resulting design criteria on composite decks with studs have concentrated on the composite action of the beam and support girder and did not consider the slab action [2,3,5]. This paper will focus on the influence of end-span studs on steel-deck-reinforced composite slabs subjected to vertical loading. This work was part of a project sponsored by the National Science Foundation on "Seismic Resistance of Composite Floor Diaphragms" [11] and the details of the work performed are described in Ref. [4].

TEST PROGRAM

To determine the influence of end-span studs on steel-deck-reinforced composite slabs, several specimens were subjected to two-point loading, as illustrated in Fig. 2. Identical slabs without end studs were tested to provide a basis for comparison. A total of fifteen specimens were cast and tested in this phase of the ISU study; however, the experience gained and the analysis procedures developed from the previous work [10] were applied in the analysis of these test results.

In general, the shear-bond mode of failure is characterized by the formation of a diagonal tension crack in the concrete at or near one of the load points, followed by a loss of bond between the steel deck and the concrete, resulting in visible slip at one end of the span, as illustrated in Fig. 3. The term shear-bond was applied to this type of failure because of the simultaneous occurrence of both the shear and bond failures. Shear-bond failure results in a loss of composite action and horizontal slippage over the region of the shear span length, L', as shown in Fig. 3. The associated end-slip indicated in the figure results from the concrete moving horizontally and overriding, or failing, the shear transfer device.

The addition of studs at the end of a shear span to provide composite action between the slab and support beam could be expected to provide restraint against the end-slip associated with shear-bond failures. Thus, the primary goals of this study included:

1. Determining the percentage of load increase for studded versus nonstudded one-way slab element specimens,

2. Determining the behavioral characteristics for the studded specimens as distinguished from nonstudded ones,

3. Developing an analysis procedure for the prediction of the ultimate load of the steel deck specimens containing studs.

Test Specimens

All fifteen specimens were nominally 3 feet wide (91.4 cm), had an overall thickness of 5-1/2 inches (14 cm), and were reinforced with

3-inch (7.6-cm)-deep steel decking of either 16- or 20-gage thickness. The fifteen specimens were divided into four groups on the basis of the out-to-out length of the specimen and the deck gage as shown in Table 1. Each group included two studded specimens together with either one or two nonstudded companion specimens for comparison. Each of the studded specimens contained typical headed studs, 3/4 inch (1.9 cm) by 4-7/8 inches (12.4 cm), placed in the down corrugation 3 inches (7.6 cm) in from the end of the specimen, as illustrated in Fig. 4. Two studs were welded through each end of the deck to 1/2-inch (1.3-cm) by 6-inch (15.2-cm) by 36-inch (91.4-cm) steel plates with the same stud and burnoff height typically used in full-scale two-way slabs. The loading apparatus was designed to provide two-point line loading, as illustrated in Fig. 2.

Test Loading and Measurements

In addition to load measurements, behavioral determinations were made for:

1. vertical deflection,

2. end-slip displacements between the deck and concrete interface,

3. slip between the deck and support plate, and

4. specimen strains.

The vertical displacements were determined at the center of the specimen and under the two load points.

The ultimate loads (P_u) recorded for each specimen are shown in Table 2. The studded specimens showed a significant increase in load-carrying capacity compared to their nonstudded companion specimens. For the 60-inch (1.52-m) shear spans, Groups I and IV, the increase was 7.7% and 24.5%, respectively. For the 18-inch (45.7-cm) shear spans, Groups II and III, the increase was 32.5% and 30.5%, respectively.

ANALYSIS

Shear-Bond Analysis

Strength relationships for the prediction of shear-bond have been proposed [7,10] from previous tests. These relationships use a combination of the basic parameters to provide a linear regression equation such as that illustrated by Fig. 5. From the figure a linear relationship can be written as

$$\frac{V_e}{bd} = \frac{m_1 \rho d}{L'} + k_1 \sqrt{f'_{ct}} \qquad (1)$$

where V_e is the experimental shear strength, b is the specimen width, d is the effective depth, ρ is the reinforcement ratio, L' is the shear span (as illustrated in Fig. 2), f'_{ct} is the concrete strength, m_1 is the slope of the regression line in Fig. 5, and k_1 is the ordinate intercept of the regression line in Fig. 5. For design purposes the experimental regression line shown in Fig. 5 is reduced to obtain new values for m and k to provide for a design equation similar to that of Eq. (1) but with reduced m and k values. Design standards based on the research performed at Iowa State University [7] and currently being developed by the American Society of Civil Engineers through their Technical Council on Codes and Standards are scheduled for publication in the near future.

Analysis of the Specimens Tested

The shear-bond analysis approach described above (as shown in Fig. 5) was also applied to the studded and nonstudded specimens tested in this series. Two approaches were tried in connection with the regression analysis of the studded specimens. One entailed a proposed regression line for the studded specimens obtained by using the same percentage increase found in the series of tests. The test results and the associated average percent increase for the studded specimens are given in Table 2. Figures 6 and 7 show the final results for the 20- and 16-gage studded specimens, respectively. The proposed regression lines illustrated in these two figures were derived by using the percentage increase in capacity found in the stud specimens as shown in Table 2. As can be seen in Figs. 6 and 7, the 18-inch (45.7-cm) shear span specimens showed a sizable load increase over the value predicted for the regression curve indicating the additional load contribution of the stud. Thus, apparently this approach does not provide a consistent prediction for all shear spans.

A second shear-bond regression approach was formulated on the basis of a regression analysis of the studded specimens themselves. The results of this analysis (Fig. 8) show that the shear-bond regression formulation approach appears feasible for the studded specimens and that the same basic parameter formulation may also work for the determination of the shear-bond strength for composite slab decks containing studs at the ends of the shear span.

BEHAVIOR

Vertical Deflections

Generally the flexural behavior exhibited three stages of stiffness, as illustrated in Fig. 9. Stage I in Fig. 9 represents the uncracked stiffness, Stage II represents the cracked stiffness, while Stage III represents the additional stiffness due to the end restraint, (i.e., due to stud shear restraint when it is present). For the deflections beyond ultimate load, the studded specimens exhibited greater ductility. Deflection behavior for the nonstudded versus studded specimens of Groups III and IV is compared in Figs. 10 and 11.

End-Slip Displacements

The studded specimens provided a significant load increase after first slip. Typical load versus end-slip behavior as illustrated by the Group IV specimens is given in Fig. 12. Note that in this figure the studded specimens were able to achieve a slightly more ductile slip behavior and slipped at a stage prior to ultimate.

Failure Mode

All of the nonstudded specimens ultimately failed because of a loss of shear-bond strength. Strains observed during testing indicated that the bottom fibers of the steel deck had yielded at the center line prior to ultimate for the specimens in Groups I, III, and IV, even though the ultimate failure mode was that of shear-bond. Large diagonal tension cracks formed under a point of loading, and the concrete section of the shear span slipped horizontally over the decking, resulting in the slip observed at the end of the specimen.

All of the studded specimens except one failed ultimately from tearing of the deck near the outer perimeter of the entire stud weld. In the one exception, longitudinal tension cracks formed in the concrete at the studs and propagated around the stud, resulting in the concrete slipping around the stud. In this case, failure of the concrete around the stud did not allow tearing of the deck to occur.

SUMMARY AND CONCLUSIONS

The tests described in this paper indicated that the addition of end studs increased the load capacity of one-way steel-deck-reinforced slabs by 8% to 33%, depending on the span and gage thickness of the deck. The nonstudded specimens ultimately failed from a loss of interfacial force in the shear span. The studded specimens ultimately failed from tearing of the deck near the stud and slippage between the concrete and steel over the length of the shear span. The increase in load capacity for the specimens containing studs was attributed to the additional stud resistance which developed as the concrete within the shear span attempted to override the deck embossments.

Two shear bond analysis approaches were attempted. The first was to find the percent of increase in the measured ultimate shear capacity and apply it to the change in the previously obtained shear-bond strength regression curves. This approach did not lead to consistent predictions for all shear spans. The second was to determine a linear regression curve for all specimens containing studs for each gage thickness of deck. This approach appears to give consistent results for the parameters plotted. Thus, a design approach similar to that for nonstudded specimens is reasonable for those slab elements containing studs at the ends of the shear spans.

APPENDIX--REFERENCES

1. American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI Standard 318-77, Detroit, Michigan: American Concrete Institute, 1977.

2. American Institute of Steel Construction, Inc., "Specification for the Design, Fabrication and Erection of Structural Steel for Building," New York: AISC, 1980.

3. Grant, J. A., Fisher, J. W., and Slutter, R. G., "Composite Beams with Formed Steel Deck," AISC Engineering Journal, American Institute of Steel Construction, First Quarter 1977, pp. 24-43.

4. Krupicka, G. L., "The Behavior and Analysis of Steel-Deck-Reinforced Concrete Slabs with End-Span Studs," Master's Thesis, Iowa State University, Ames, Iowa, 1979, 97 pp.

5. Ollgaard, J. G., Slutter, R. G., and Fisher, J. W., "Shear Strength of Stud Connectors in Lightweight and Normal-Weight Concrete," AISC Engineering Journal, American Institute of Steel Construction, 8, No. 2, April 1971, pp. 55-64.

6. Porter, M. L., and Ekberg, C. E., Jr., "Behavior of Steel-Deck Reinforced Slabs," Journal of the Structural Division, ASCE, Vol. 103, No. ST3, Proc. Paper 12826, March 1977, pp. 663-677.

7. Porter, M. L., and Ekberg, C. E., Jr., "Compendium of ISU Research Conducted on Cold-Formed Steel-Deck-Reinforced Slab Systems," Bulletin 200, Engineering Research Institute, Iowa State University, December 1978.

8. Porter, M. L., and Ekberg, C. E., Jr., "Design Recommendations for Steel Deck Floor Slabs," Journal of the Structural Division, ASCE, Vol. 102, No. ST11, Proc. Paper 12528, November 1976, pp. 2121-2136.

9. Porter, M. L., and Ekberg, C. E., Jr., "Design vs Test Results for Steel Deck Floor Slabs," Proceedings of Third Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Rolla, October 1975, pp. 293-812.

10. Porter, M. L., Ekberg, C. E., Jr., Greimann, L. F., and Elleby, H. A., "Shear-Bond Analysis of Steel-Deck-Reinforced Slabs," Journal of the Structural Division, ASCE, Vol. 102, No. ST12, Proc. Paper 12611, December 1976, pp. 2255-2268.

11. Porter, M. L., and Greimann, L. F., "Seismic Resistance of Composite Floor Diaphragms," ISU-ERI-Ames-80133, Iowa State University, Ames, Iowa, Final Report submitted to National Science Foundation, May 1980, 174 pp.

APPENDIX--NOTATION

A_s Cross-sectional area of steel deck or area of negative moment reinforcing steel where used as tension reinforcement

b Unit width of slab

b_d Width of composite test slab

d Effective slab depth (distance from extreme concrete compression fiber to centroidal axis of the full cross section of the steel deck)

f'_{ct} Compressive test cylinder strength at time of slab testing

k Ordinate intercept of reduced shear-bond line (see Fig. 5)

k_1 Ordinate intercept of shear-bond line (see Fig. 5)

L Length of span

L' Length of shear span; for uniform load, L' = one quarter of the span

m Slope of reduced shear-bond line (see Fig. 4)

m_1 Slope of shear-bond line (see Fig. 4)

P_e Maximum applied experimental slab load at failure obtained from laboratory tests (includes weight of loading system but not weight of slab)

V_e Maximum experimental shear at failure obtained from laboratory tests (not including weight of slab)

ρ Reinforcement ratio of steel deck area to effective concrete area, A_s/bd

LIST OF TABLES

Table 1. Specimen groups for vertical loading.

Group	Specimens	Specimen Length (inches)	Deck Gage	Steel thickness (inches)
I	1-4*	184	20	0.0337
II	5-8	92	20	0.0337
III	9-12	73	16	0.0595
IV	13-15	184	16	0.0530

*Refers to slab number and type in Table 2.

Table 2. Vertical loading test results.

Slab Number and Type	Span Length, L_c (inches)	Shear Span Length, L' (inches)	Ultimate Load, P_u (kips)	Average % Increase
1 - nonstudded	178	60	6.47	
2 - nonstudded	178	60	6.11	
3 - studded	178	60	6.58	7.7
4 - studded	178	60	7.00	
5 - nonstudded	86	18	17.73	
6 - nonstudded	86	18	18.73	
7 - studded	86	18	28.25	32.5
8 - studded	86	18	25.75	
9 - nonstudded	67	18	28.75	
10 - nonstudded	67	18	28.50	
11 - studded	67	18	40.75	30.5
12 - studded	67	18	41.50	
13 - nonstudded	178	60	9.06	
14 - studded	178	60	12.18	24.5
15 - studded	178	60	11.68	

1 inch = 2.54 cm

1 kip = 4.45 kN

LIST OF FIGURES

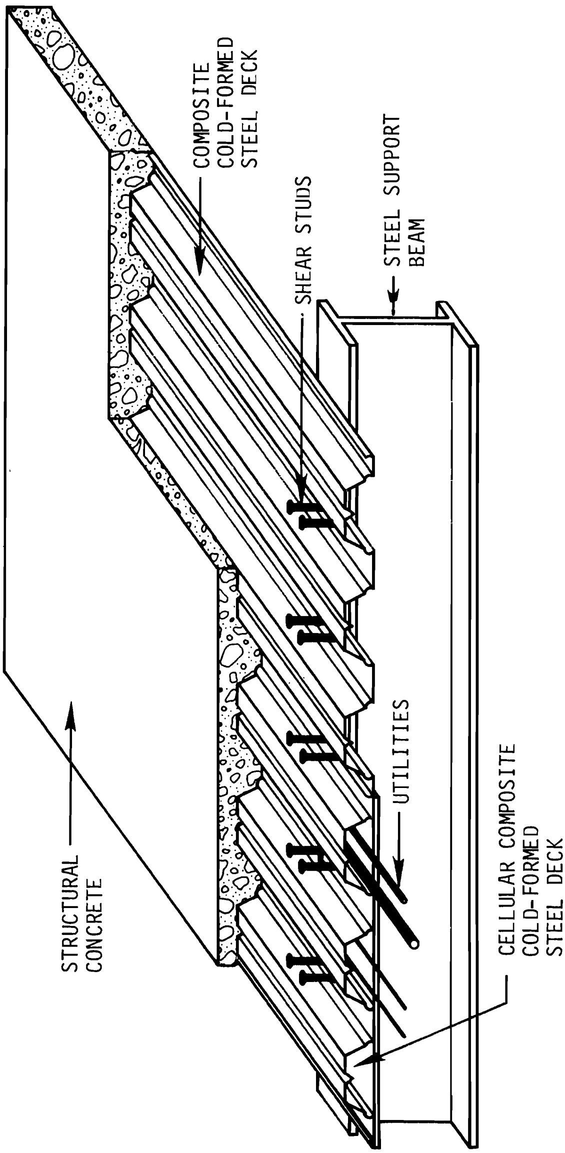

Fig. 1. Typical construction utilizing cold-formed steel decking with composite support beams.

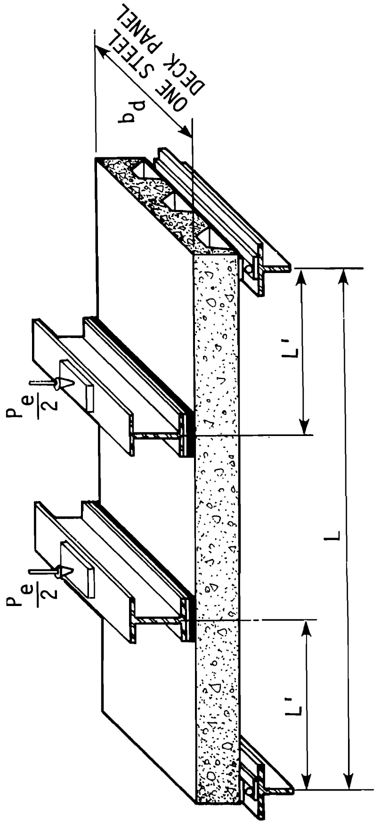

Fig. 2. Typical arrangement for testing one-way slab elements.

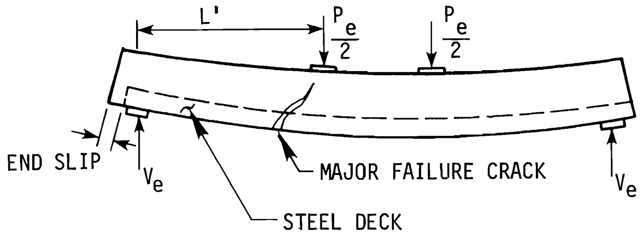

Fig. 3. Typical shear-bond failure.

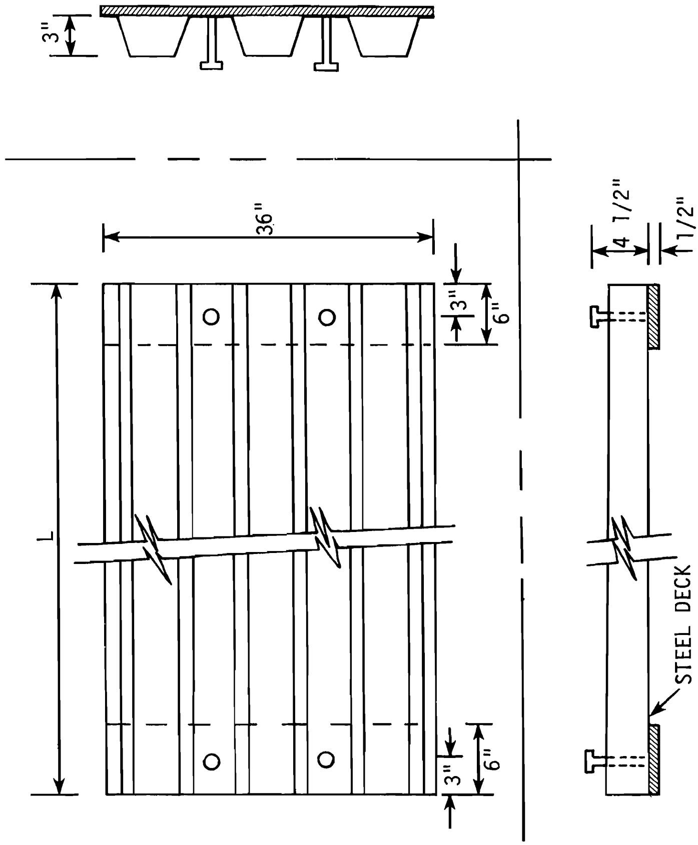

Fig. 4. Studded specimen details prior to casting

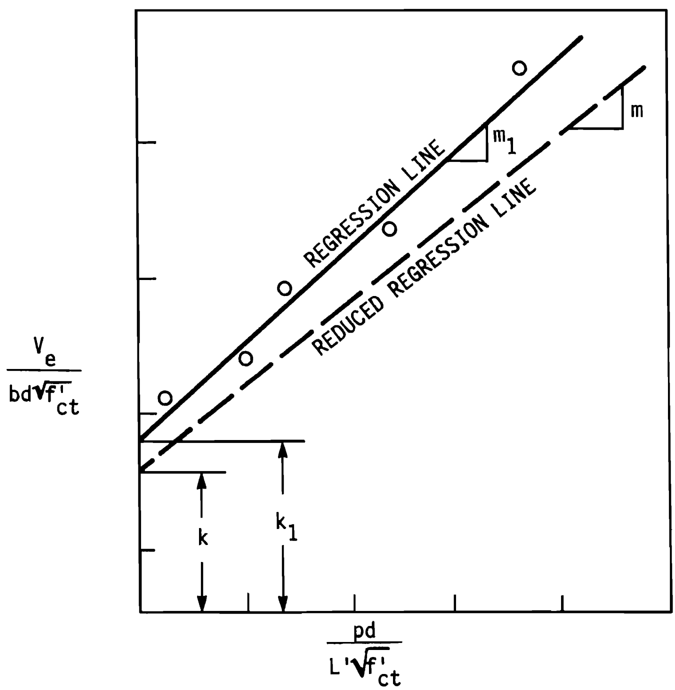

Fig. 5. Typical shear-bond plot showing the reduced evaluation of m and k.

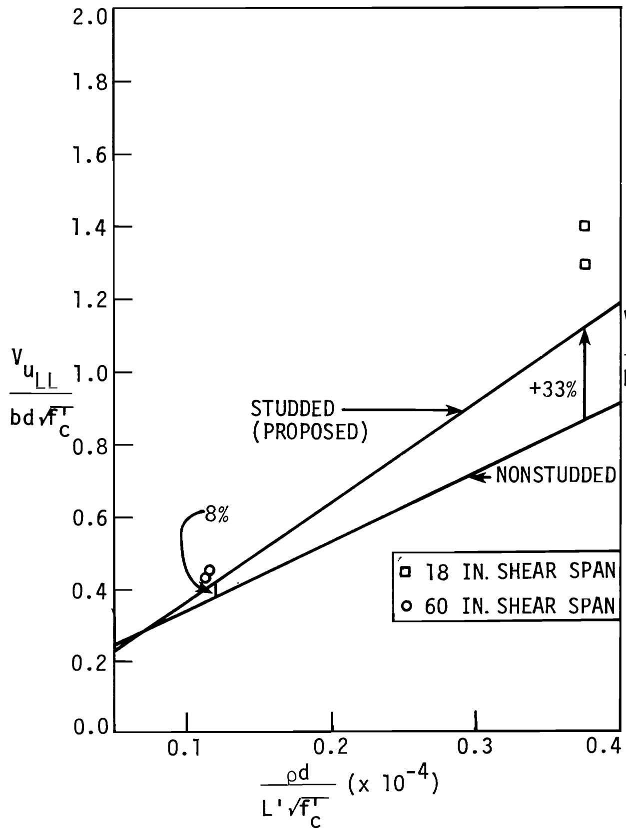

Fig. 6. Proposed studded specimen curve, 20-gage.

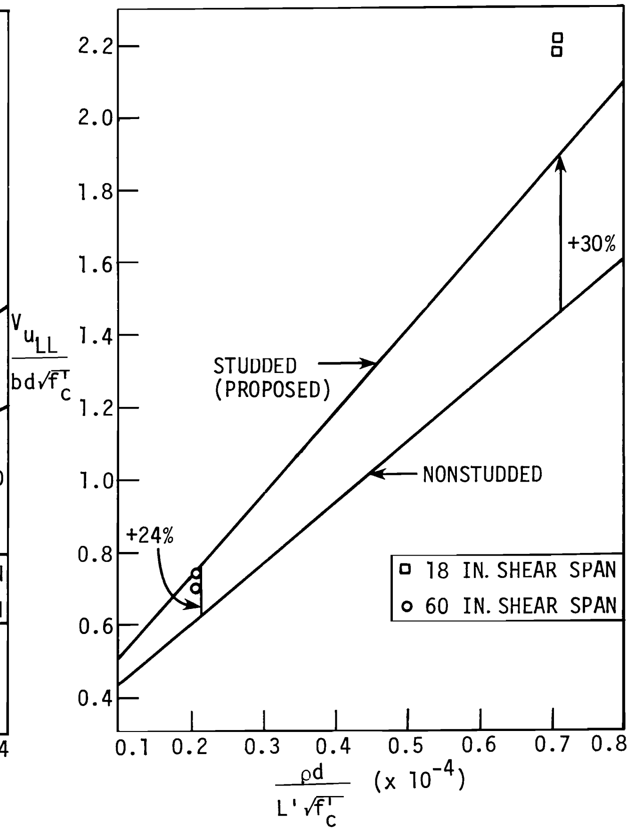

Fig. 7. Proposed studded specimen curve, 16-gage.

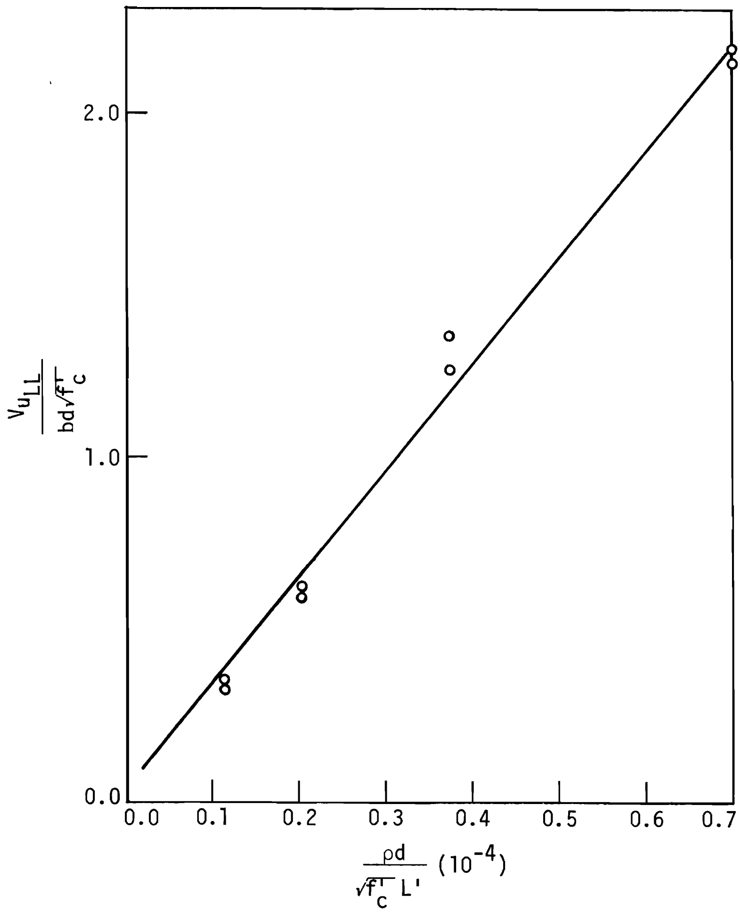

Fig. 8. Plot of studded specimen results, gages combined.

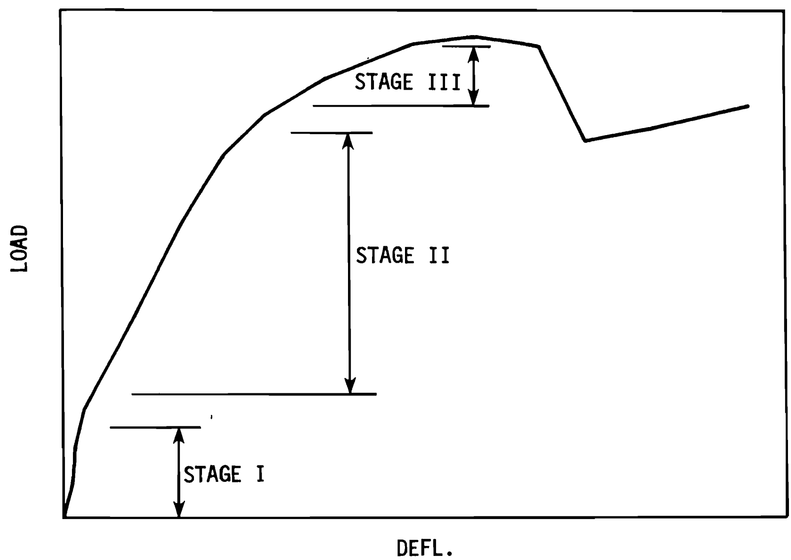

Fig. 9. Three stages of bending stiffness.

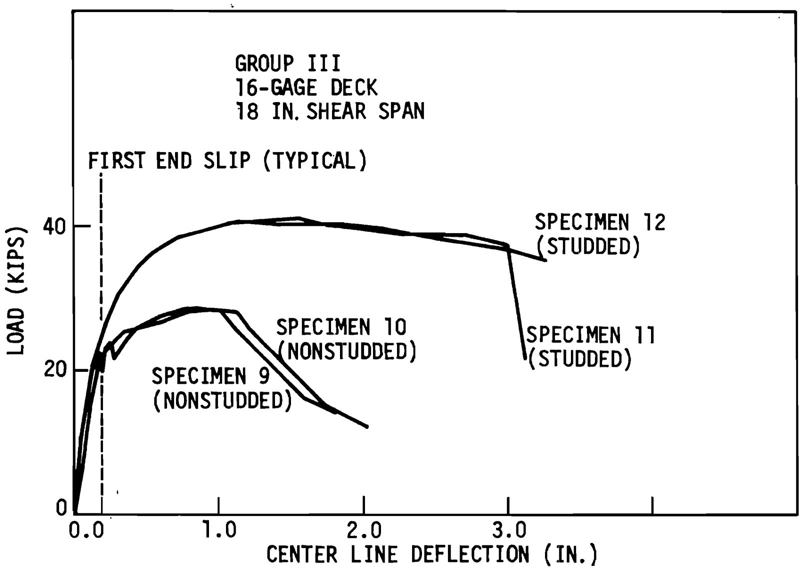

Fig. 10. Load vs. center line deflection diagram, Group III.

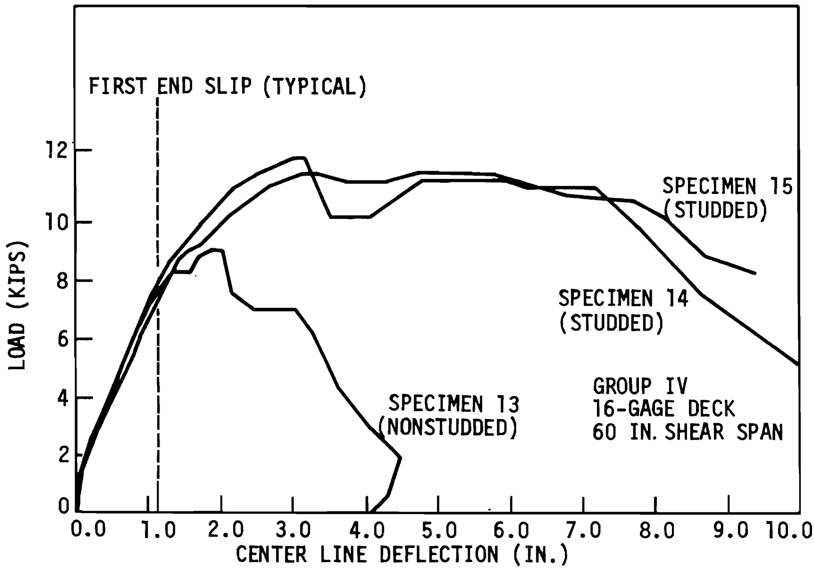

Fig. 11. Load vs. center line deflection diagram, Group IV.

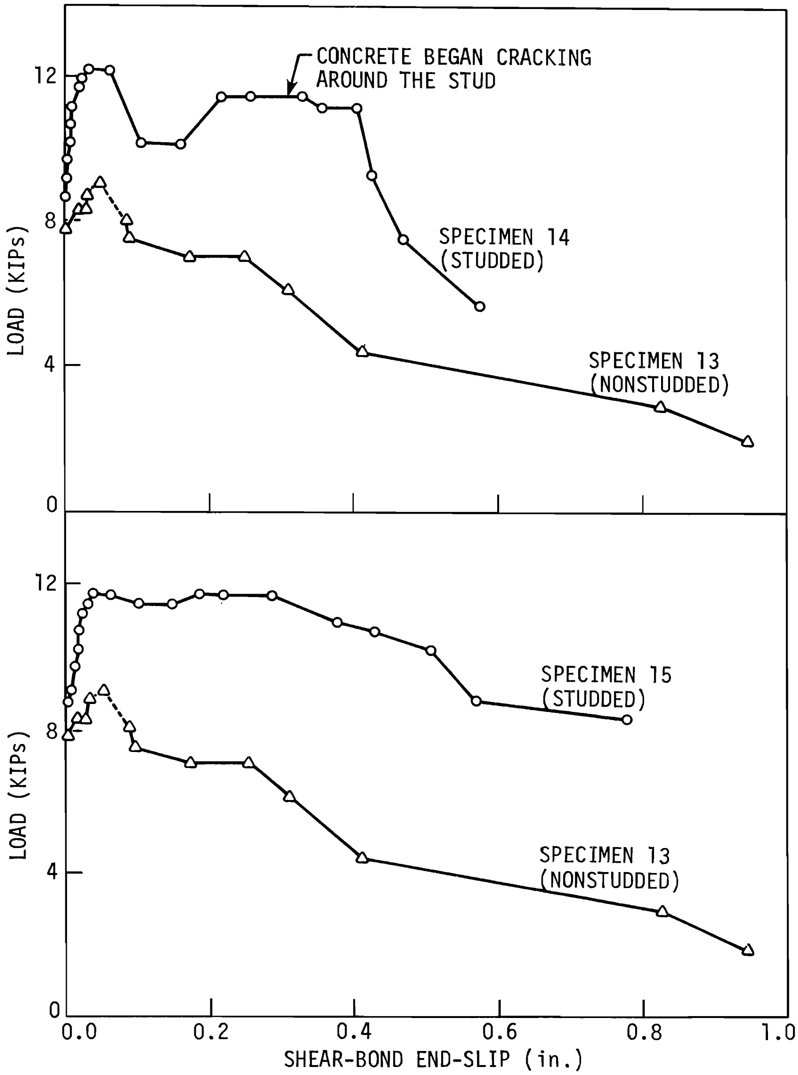

Fig. 12. Load vs. shear-bond end-slip, Group IV.